Chapter 7
Design of Initial Support

7-1. Design of Initial Ground Support

a. Initial ground support is installed shortly after excavation in order to make the underground opening safe until permanent support is installed. The initial ground support may also function as the permanent ground support or as a part of the permanent ground support system. The initial ground support must be selected in view of both its temporary and permanent functions.

b. Because of the variability of geologic materials, initial ground support systems are usually not subject to rigorous design but are selected on the basis of a variety of rules. There are three basic methodologies employed in selecting initial ground support, and one or more of these approaches should be used:

- Empirical rules constructed from experience records of satisfactory past performance.

- Theoretical or semitheoretical analysis methods, based on one or more postulated modes of behavior.

- The fundamental approach, involving a definition of potential modes of failure and a selection or design of components to resist these modes of failure.

EM 1110-1-2907 (Rock Reinforcement) and EM 1110-2-2005 (Standard Practice for Shotcrete) provide additional details on these types of ground support.

7-2. Empirical Selection of Ground Support

In past centuries, ground support was always selected empirically. The miner estimated, based on his experience, what timbering was required, and if the timbering failed it was rebuilt stronger. Written rules for selecting ground support were first formulated by Terzaghi (1946). The development of the RQD as a means to describe the character or quality of the rock mass led to correlations between RQD and Terzaghi's rock loads. This development also led to independent ground support recommendations based on RQD. The RQD is also of the basis of two other rock mass characterization schemes used for initial ground support selection, the Geomechanics Classification (Rock Structure Rating (RMR) scheme, Bieniawski 1979), and the Norwegian Geotechnical Institute's Q-system (Barton, Lien, and Lunde 1974). Another classification and ground support selection scheme, the Rock Structure Rating (RSR, Wickham, Tiedemann, and Skinner 1974), is also used.

a. Terzaghi's rock loads and the RQD.

(1) Terzaghi estimated rock loads on steel ribs based on verbal descriptions of the rock mass characteristics. He described the vertical and side loads on the ribs in terms of the height of a loosened mass weighing on the steel rib. The height is a multiple of the width of the tunnel or of the width plus the height. The rock mass descriptions are discussed in Section 3-3. Deere et al. (1970) correlated Terzaghi's rock loads with approximate RQD values and approximate fracture spacings as shown in Table 7-1, and also presented separate ground support recommendations for tunnels excavated conventionally and by TBM as shown in Table 7-2.

(2) Terzaghi's rock load estimates were derived from an experience record that included tunnels excavated by blasting methods and supported by steel ribs or timbers. Ground disturbance and loosening occur due to the blasting prior to installation of initial ground support, and the timber blocking used with ribs permits some displacement of the rock mass. Terzaghi's rock loads generally should not be used in conjunction with methods of excavation and support that tend to minimize rock mass disturbance and loosening, such as excavation on TBM and immediate ground support using shotcrete and dowels. The Deere et al. recommendations are still sound and reasonable, but are now used mainly as a check on other empirical methods.

b. Rock Structure Rating (RSR).

(1) The Rock Structure Rating system was devised by Wickham, Tiedeman, and Skinner in 1972. It was the first published, numerical rating of a rock mass that takes into account a number of geologic parameters and produces a numerical rock load estimate. The geologic parameters considered include the following:

- Rock type.

- Joint pattern (average joint spacing).

- Joint orientations (dip and strike).

- Type of discontinuities.

- Major faults, shears, and folds.

Table 7-1
Terzaghi's Rock Load Classifications as Modified by Deere et al. 1970

Fracture spacing (cm)	RQD (%)	Rock condition	Rock load, H_p Initial	Rock load, H_p Final	Remarks	
		1. Hard and intact	0	0	Generally no side pressure, erratic load changes from point to point	Lining only if spalling or popping
50	98 95	2. Hard stratified or schistose	0	$0.25B$		Spalling common
	90	3. Massive moderately jointed	0	$0.5B$		Side pressure if strata inclined, some spalling
20		4. Moderately blocky and seamy	0	$0.25B$ $0.35C$		
10	75 50	5. Very blocky, seamy and shattered	0 to $0.6C$	$0.35B$ $1.1C$	Little or no side pressure	
5	25 10 2	6. Completely crushed		$1.1C$	Considerable side pressure. If seepage, continuous support	
2		7. Gravel and sand	$0.54C$ to $1.2C$	$0.62C$ to $1.38C$	Dense	
					Side Pressure $Ph = 0.3\gamma(0.5H_t + H_p)$	
			$0.94C$ to $1.2C$	$1.08C$ to $1.38C$	Loose	
Weak and Coherent		8. Squeezing, moderate depth		$1.C$ to $2.1C$	Heavy side pressure, continuous support required	
Weak and Coherent		9. Squeezing, great depth		$2.1C$ to $4.5C$		
		10. Swelling		up to 75 m (250 ft)	Use circular support. In extreme cases: yielding support	

Notes:
1. For rock classes 4, 5, 6, 7, when above groundwater level, reduce loads by 50 percent.
2. B is tunnel width; $C = B + H_t$ = width + height of tunnel.
3. γ = density of medium.

Table 7-2
Support Recommendations for Tunnels in Rock (6 m to 12 m diam) Based on RQD (after Deere et al. 1970)

Rock Quality	Tunneling Method	Alternative Support Systems Steel Sets[3]	Rockbolts[3]	Shotcrete
Excellent[1] RQD>90	Boring machine	None to occasional light set. Rock load (0.0-0.2) B	None to occasional	None to occasional local application
	Conventional	None to occasional light set. Rock load (0.0-0.3) B	None to occasional	None to occasional local application 2 to 3 in.
Good[1] 75<RQD<90	Boring machine	Occasional light sets to pattern on 5- to 6-ft center. Rock load (0.0 to 0.4)B	Occasional to pattern on 5- to 6-ft centers	None to occasional local application 2 to 3 in.
	Conventional	Light sets 5- to 6-ft center. Rock load (0.3 to 0.6)B	Pattern, 5- to 6-ft centers	Occasional local application 2 to 3 in.
Fair 50<RQD<75	Boring machine	Light to medium sets, 5- to 6-ft center. Rock load (0.4-1.0)B	Pattern, 4- to 6-ft center	2- to 4-in. crown
	Conventional	Light to medium sets, 4- to 5-ft center. Rock load (0.6-1.3)B	Pattern, 3- to 5-ft center	4-in. or more crown and sides
Poor[2] 25<RQD<50	Boring machine	Medium circular sets on 3- to 4-ft center. Rock load (1.0-1.6)B	Pattern, 3- to 5-ft center	4 to 6 in. on crown and sides. Combine with bolts.
	Conventional	Medium to heavy circular sets on 2- to 4-ft center. Rock load (1.3-2.0)B	Pattern, 2- to 4-ft center	6 in. or more on crown and sides. Combine with bolts.
Very poor[3] RQD<25 (Excluding squeezing or swelling ground)	Boring machine	Medium to heavy circular sets on 2-ft center. Rock load (1.6 to 2.2)B	Pattern, 2- to 3-ft center	6 in. or more on whole section. Combine with medium sets.
	Conventional	Heavy circular sets on 2-ft center. Rock load (1.6 to 2.2)B	Pattern, 3-ft center	6 in. or more on whole section. Combine with medium sets.
Very poor[3] (Squeezing or swelling)	Boring machine	Very heavy circular sets on 2-ft center. Rock load up to 250 ft.	Pattern, 2- to 3-ft center	6 in. or more on whole section. Combine with heavy sets.
	Conventional	Very heavy circular sets on 2-ft center. Rock load up to 250 ft.	Pattern, 2- to 3-ft center	6 in. or more on whole section Combine with heavy sets.

Notes:
[1] In good and excellent rock, the support requirement will be, in general, minimal but will be dependent upon joint geometry, tunnel diameter, and relative orientations of joints and tunnel.
[2] Lagging requirements will usually be zero in excellent rock and will range from up to 25 percent in good rock to 100 percent in very poor rock.
[3] Mesh requirements usually will be zero in excellent rock and will range from occasional mesh (or strips) in good rock to 100-percent mesh in very poor rock.
[4] B = tunnel width.

- Rock material properties.
- Weathering and alteration.

(2) Some of these are combined in various ways. The construction parameters are size of tunnel, direction of drive (relative to discontinuities), and method of excavation. All of these parameters are combined as shown in Table 7-3; the RSR value is the sum of parameters A, B, and C. With the assumption that TBM excavation causes less disturbance, the RSR value is adjusted by the factor shown on Figure 7-1 as a function of tunnel size.

(3) Predicted tunnel arch rock loads in kips per square foot as a function of RSR and tunnel width or diameter are shown on Figure 7-2.

Table 7-3
Rock Structure Rating - Parameter A: General Area Geology (after Wickham et al. 1974)

	Basic Rock Type				Geological Structure			
	Hard	Med.	Soft	Decomp.	Massive	Slightly faulted or folded	Moderately faulted or folded	Intensely faulted or folded
Igneous	1	2	3	4				
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4				
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

Rock Structure Rating - Parameter B: Joint Pattern, Direction of Drive (after Wickham et al. 1974)

Average joint spacing	Strike ⊥ to axis					Strike ∥ to axis		
	Direction of drive					Direction of drive		
	Both	With dip		Against Dip		Both		
	Dip of prominent joints[1]					Dip of prominent joints[1]		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed < 2 in.	9	11	13	10	12	9	9	7
2. Closely jointed 2-6 in.	13	16	19	15	17	14	14	11
3. Moderately jointed 6-12 in.	23	24	28	19	22	23	23	19
4. Moderate to blocky 1-2 ft	30	32	36	25	28	30	28	24
5. Blocky to massive 2-4 ft	36	38	40	33	35	36	34	28
6. Massive > 4 ft	40	43	45	37	40	40	38	34

Rock Structure Rating - Parameter C: Groundwater, Joint Condition (after Wickham et al. 1974)

Anticipated water inflow (gpm/1,000 ft)	Sum of parameters $A + B$					
	13-44			45-75		
	Joint condition[2]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight < 200 gpm	19	15	9	23	19	14
Moderate 200-1,000 gpm	15	11	7	21	16	12
Heavy > 1,000 gpm	10	8	6	18	14	10

[1] Dip: flat: 0-20 deg; dipping: 20-50 deg; and vertical: 50-90 deg.
[2] Joint condition: Good = tight or cemented; Fair = slightly weathered or altered; Poor = severely weathered, altered, or open.

(4) The RSR database consists of 190 tunnel cross sections, of which only three were shotcrete supported and 14 rock bolt supported; therefore, the database only supports rock load recommendations for steel ribs.

c. *Geomechanics Classification (RMR System).*

(1) This system, developed by Bieniawski (1979), uses the following six parameters:

- Uniaxial compressive strength of rock.
- RQD.
- Spacing of discontinuities.
- Condition of discontinuities.
- Groundwater condition.
- Orientation of discontinuities.

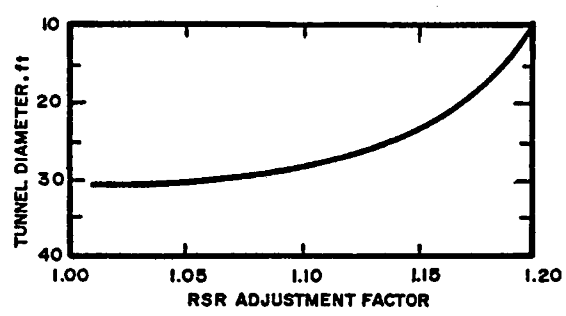


Figure 7-1. RSR adjustment factor for TBM excavation

The components of this classification system are shown in Table 7-4. Part A of this table shows the five basic parameters and their ranges as dependent on the rock mass condition. Together, the rating numbers for the five parameters add up to the basic RMR value. Part B gives a rating adjustment based on the orientation of the discontinuities relative to the tunnel orientation. The effect of strike and dip on tunneling is shown in Table 7-5. Part C of Table 7-4 shows the general classification of the rock mass based on RMR, ranging from very good to very poor rock. Part D presents some numerical predictions of stand-up time, rock mass cohesion, and friction based on RMR. Unal (1983) presented the following equation for the ground load, measured as the rock load height:

$$H_b = (1 - RMR/100)\ B$$

where B is the tunnel width. Recommendations for excavation and support for a 10-m-wide tunnel excavated by blasting are presented in Table 7-6.

(2) Other correlations using RMR have been developed. Figure 7-3 shows a correlation between RMR and the in situ modulus of deformation of the rock mass. Serafin and Pereira (1983) produced a different correlation, applicable also for RMR < 50:

$$E_M = 10\ (RMR/40 - 0.25)$$

(3) The RMR system is based on a set of case histories of relatively large tunnels excavated using blasting. Ground support components include rock bolts (dowels), shotcrete, wire mesh, and for the two poorest rock classes, steel ribs. The system is well suited for such conditions but not for TBM-driven tunnels, where rock damage is less and where immediate shotcrete application may not be feasible.

d. The Q-System for rock mass classification.

(1) The NGI Q-System (Barton, Lien and Lunde 1974) is generally considered the most elaborate and the most detailed rock mass classification system for ground support in underground works. The value of the rock quality index Q is determined by

$$Q = (RQD/J_n)\ (J_r/J_a)\ (J_w/SRF)$$

where

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

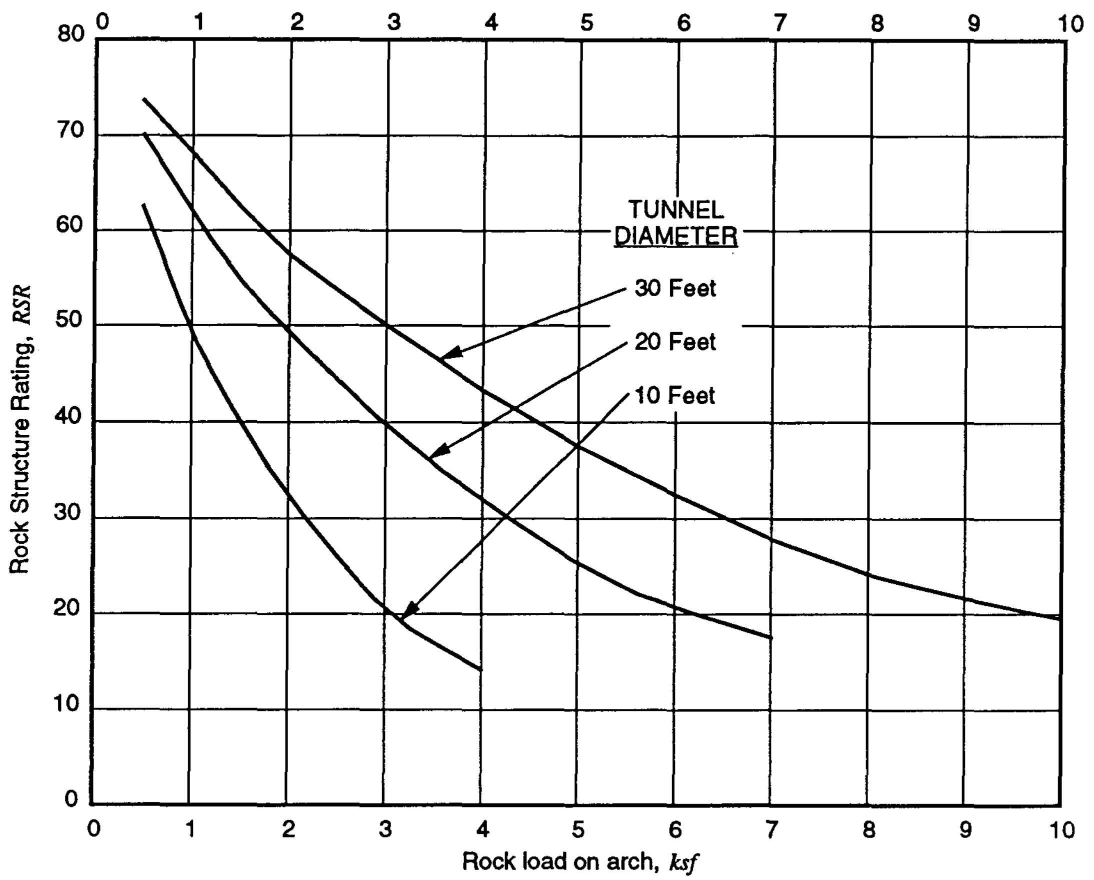
J_w = joint water reduction factor

SRF = stress reduction factor

The numerical values of these numbers are determined as described in Table 7-7.

(2) To relate the Q-value to ground support requirements, an equivalent dimension is defined as the width of the underground opening, divided by the excavation support ratio (ESR). The value of the ESR depends on the ultimate use of the underground opening and the time of exposure; the following values of ESR are recommended:

- ESR = 3-5 for temporary mine openings.
- ESR = 2-2.5 for vertical shafts (highest for circular).
- ESR = 1.6 for permanent mine openings, hydropower water tunnels (except high-pressure tunnels), and temporary works, including tunnels where a final lining is later placed.
- ESR = 1.3 for minor traffic tunnels, surge chambers, access tunnels.
- ESR = 1.0 for most civil works, including power stations, major traffic tunnels, water pressure tunnels, intersections of tunnels, and portals.



Correlation of Rock Structure Rating to Rock Load and Tunnel Diameter

Tunnel Diameter (D)	(Wr) Rock Load on Tunnel Arch (k/sq ft)											
	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
	Corresponding Values of Rock Structure Ratings (RSR)											
10'	62.5	49.9	40.2	32.7	21.6	13.8						
12'	65.0	53.7	44.7	37.5	26.6	18.7						
14'	66.9	56.6	48.3	41.4	30.8	22.9	16.8					
16'	68.3	59.0	51.2	44.7	34.4	26.6	20.4	15.5				
18'	69.5	61.0	53.7	47.6	37.6	29.9	23.8	18.8				
20'	70.4	62.5	55.7	49.9	40.2	32.7	26.6	21.6	17.4			
22'	71.3	63.9	57.5	51.9	42.7	35.3	29.3	24.3	20.1	16.4		
24'	72.0	65.0	59.0	53.7	44.7	37.5	31.5	26.6	22.3	18.7		
26'	72.6	66.1	60.3	55.3	46.7	39.6	33.8	28.8	24.6	20.9	17.7	
28'	73.0	66.9	61.5	56.6	48.3	41.4	35.7	30.8	26.6	22.9	19.7	16.8
30'	73.4	67.7	62.4	57.8	49.8	43.1	37.4	32.6	28.4	24.7	21.5	18.6

Figure 7-2. Tunnel arch load as a function of RSR and tunnel diameter

Table 7-4
Geomechanics Classification of Jointed Rock Masses

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred		
			>250 MPa	100-150 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90-100%	75-90%	50-75%	25-50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6-2 m	200-600 mm	60-200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces. Not continuous. No separation. Unweathered wall rock.	Slightly rough surfaces. Separation <1 mm. Slightly weathered walls.	Slightly rough surfaces. Separation < 1 mm. Highly weathered walls.	Slickensided surfaces OR Gouge < 5 mm thick. Separation 1-5 mm. Continuous.	Soft gouge > 5 mm thick OR Separation > 5 mm. Continuous.		
	Rating		30	25	20	10	0		
5	Ground-water	Inflow per 10 m tunnel length	None	<10 L/min	10-25 L/min	25-125 L/min	>125 L/min		
		Ratio: joint water pressure / major principal stress	OR 0	OR 0.0-0.1	OR 0.1-0.2	OR 0.2-0.5	OR >0.5		
		General conditions	OR Completely dry	OR Damp	OR Wet	OR Dripping	OR Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations and dips		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	41 ← 21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15-m span	6 months for 8-m span	1 week for 5-m span	10 hr for 2.5-m span	30 min for 1-m span
Cohesion of the rock mass	>400 kPa	300-400 kPa	200-300 kPa	100-200 kPa	<100 kPa
Friction angle of the rock mass	>45°	35-45°	25-45°	15-25°	<15°

Table 7-5
Effect of Discontinuity Strike and Dip Orientations in Tunneling

Strike perpendicular to tunnel axis Drive with dip			
Dip 45-90°	Dip 20-45°	Dip 45-90°	Dip 20-45°
Strike parallel to tunnel axis			Irrespective of strike
Dip 20-45°	Dip 45-90°		Dip 0-20°
Fair	Very Unfavorable		Fair

Table 7-6
Geomechanics Classification Guide for Excavation and Support in Rock Tunnels After Bieniawski (1979)

SHAPE: HORSESHOE; WIDTH: 10 M; VERTICAL STRESS: BELOW 25 MPa; CONSTRUCTION: DRILLING AND BLASTING

Rock Mass Class	Excavation	Rock Bolts (20 mm diam., fully bonded)	Shotcrete	Steel Sets
Very good rock, I RMR:81-100	Full face 3-m advance.	Generally no support required except for occasional spot bolting.		
Good rock, II RMR:61-80	Full face 1.0- to 1.5-m advance. Complete support 20 m from face.	Locally bolts in crown 3 mm long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None
Fair rock, III RMR:41-60	Top heading and bench 1.5- to 3-m advance in top heading. Commerce support after each	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	None
Poor rock, IV RMR:21-40	Top heading and bench 1.0- to 1.5-m advance in top heading. Install support concurrently with excavation 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
Very poor rock, V RMR: <20	Multiple drifts. 0.5- to 1.5-m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

- ESR = 0.8 for underground railroad stations, sports arenas, and similar public areas.

(3) For application to initial support, where a final lining is placed later, multiply the ESR value by 1.5. The following correlations apply, albeit with considerable variation:

- Maximum unsupported span = 2 ESR $Q^{0.4}$ (m).

- Permanent support pressure, with three or more joint sets: $P = 2.0\ Q^{-1/3}/Jr$.

- Permanent support pressure, with less than three joint sets: $P = 2.0\ J_n^{1/2} Q^{-1/3}/3J_r$.

(4) Barton, Lien, and Lunde (1974) provide 38 support categories (see Figure 7-4) with detailed support recommendations, as enumerated in the annotated Table 7-8.

(5) With all of the commentaries accompanying the tables, the Q-system works very much like an expert system. A careful examination of all the commentaries reveals that the system incorporates features of rock behavior not entirely evident from the basic parameters. This adds to the flexibility and range of application of the system.

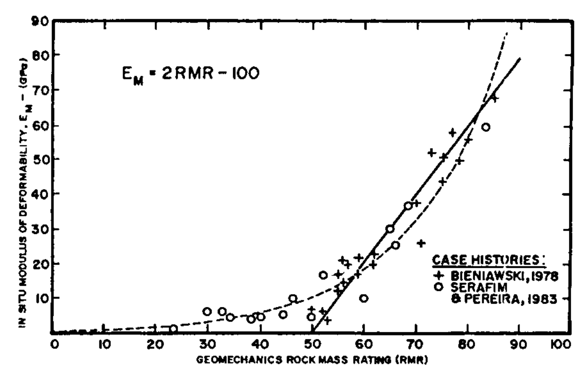


Figure 7-3. Correlation between in situ modulus of deformation and RMR

(6) The Q-system is derived from a database of underground openings excavated by blasting and supported by rock bolts (tensioned and untensioned), shotcrete, wire and chain-link mesh, and cast-in-place concrete arches. For TBM-driven tunnels, it is recommended that the Q-value should be increased by a factor of 5.0.

e. Restrictions in the use of empirical ground support selection systems.

(1) The empirical methods of ground support selection provide a means to select a ground support scheme based on facts that can be determined from explorations, observations, and testing. They are far from perfect and can sometimes lead to the selection of inadequate ground support. It is therefore necessary to examine the available rock mass information to determine if there are any applicable failure modes not addressed by the empirical systems.

(2) A major flaw of all the empirical systems is that they lead the user directly from the geologic characterization of the rock mass to a recommended ground support without the consideration of possible failure modes. A number of potential modes of failure are not covered by some or all of the empirical methods and must be considered independently, including the following:

- Failure due to weathering or deterioration of the rock mass.

- Failure caused by moving water (erosion, dissolution, excessive leakage, etc.).

- Failure due to corrosion of ground support components.

- Failure due to squeezing and swelling conditions.

- Failure due to overstress in massive rock.

(3) The empirical systems are largely based on blasted tunnels and produce ground support recommendations that are a function of the age of the empirical system. System recommendations should be reinterpreted based on current methods of excavation. For example, TBM tunneling produces a favorable tunnel shape and a minimum of ground disturbance; however, the application of shotcrete close to the tunnel face is difficult. Therefore, substitutes for shotcrete, including dowels with wire mesh, ribs with wire mesh, or precast segments, must be applied.

(4) Similarly, new ground support methods and components must be considered. For example, the use of steel fiber reinforced shotcrete, friction dowels, lattice girders, or segmental concrete linings are not incorporated in the empirical systems.

7-3. Theoretical and Semitheoretical Methods

Most theoretical methods of design for rock bolts, dowels, or shotcrete are based on certain assumptions regarding the configuration of discontinuities.

a. Rock bolt analyses.

(1) The simplest methods of rock bolt analysis are the wedge analyses, where the stability of a wedge is analyzed using two- or three-dimensional equilibrium equations. Examples are shown in Figure 7-5. These types of analysis are useful when directions of discontinuities are known and can show which wedges are potentially unstable and indicate the appropriate orientation of bolts or dowels for their support.

(2) For a flat roof in a horizontally layered rock (Figure 7-6), Lang and Bischoff (1982) developed an analysis to show the effect of rock bolts. If the rock bolts are tensioned, either by active tensioning or passively by ground movements, a horizontal compressive stress develops within the zone of the bolts. This enables the beam consisting of the layers of rock tied together to carry a moment, and the edge of the beam to carry a shear load. Thus, the reinforced rock stays suspended. In a similar manner, bolts installed around an arch will increase the

Table 7-7
Input Value to Estimate of Q

1. ROCK QUALITY DESIGNATION (RQD)

A. Very poor	0 - 25
B. Poor 25 - 50	
C. Fair	50 - 75
D. Good	75 - 90
E. Excellent	90 - 100

Note: (i) Where RQD is reported or measured as <10 (including 0), a nominal value of 10 is used to evaluate Q in equation (1)
(ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate

2. JOINT SET NUMBER (J_n)

A.	Massive, none or few joints	0.5 - 1.0
B.	One joint set	2
C.	One joint set plus random	3
D.	Two joint sets	4
E.	Two joint sets plus random	6
F.	Three joint sets	9
G.	Three joint sets plus random	12
H.	Four or more joint sets, random, heavily jointed, "sugar cube," etc.	15
J.	Crushed rock, earthlike	20

Note: (i) For intersections use (3.0 x J_n)

Note: (ii) For portals use (2.0 J_n)

3. JOINT ROUGHNESS NUMBER

(a) *Rock wall contact* and
(b) *Rock wall contact before* 100-*mm shear* (J_r)

A.	Discontinuous joints	4
B.	Rough or irregular, undulating	3
C.	Smooth, undulating random	2
D.	Slickensided, undulating	1.5
E.	Rough or irregular, planar	1.5
F.	Smooth, planar	1.0
G.	Slickensided, planar	0.5

Note: (i) Descriptions refer to small-scale features and intermediate scale features, in that order.

(c) *No rock wall contact when sheared*

H.	Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J.	Sandy, gravelly, or crushed, some thick enough to prevent rock wall contact	1.0

Note: (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
(iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are orientated for minimum strength

(Sheet 1 of 3)

Table 7-7. (Continued)

4.	<u>JOINT ALTERATION NUMBER</u>	(J_a)	ϕ_r
A.	Tightly healed, hard, nonsoftening, impermeable filling, i.e., quartz or epidote	0.75	(-)
B.	Unaltered joint walls, surface staining only.	1.0	(25-35°)
C.	Slightly altered joint walls. Nonsoftening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	(25-30°)
D.	Silty- or sandy-clay coatings, small clay fraction (nonsoft.)	3.0	(20-25°)
E.	Softening or low-friction clay mineral coatings, i.e., kaolinite or mica. Also, chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays..	4.0	(8-16°)
	(b) *Rock wall contact before 100-mm shear*		
F.	Sandy particles, clay-free disintegrated rock, etc..	4.0	(25-30°)
G.	Strongly overconsolidated nonsoftening clay mineral fillings (continuous, but <5-mm thickness).	6.0	(16-24°)
H.	Medium or low overconsolidation, softening, clay-mineral fillings (continuous but <5-mm thickness).	8.0	(12-16°)
J.	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5-mm thickness) Value of Ja depends on percent of swelling clay-size particles and access to water, etc.	8 - 12	(6-12°)
	(c) *No rock wall contact when sheared*		
K.L. M.	Zones or bands of disintegrated or crushed rock and clay (see G,H,J for description of clay condition)	6, 8 or 8-12	(6-24°)
N.	Zones or bands of silty- or sandy-clay, small clay fraction (nonsoftening).	5.0	(-)
O.P. R.	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition	10, 13, or 13-20	(6-24°)
5.	<u>JOINT WATER REDUCTION FACTOR</u>	(J_w)	Approx. water pres. (kPa^2)
A.	Dry excavations or minor inflow, i.e., <5 ℓ/min. locally	1.0	<100
B.	Medium inflow or pressure, occasional outwash of joint fillings	0.66	100-250
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	250-1,000
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	250-1,000
E.	Exceptionally high inflow or water pressure at blasting, decaying with time.	0.2-0.1	>1,000
F.	Exceptionally high inflow or water pressure continuing without noticeable decay.	0.1-0.05	>1,000

Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
(ii) Special problems caused by ice formation are not considered.

(Sheet 2 of 3)

Table 7-7 (Concluded)

6. STRESS REDUCTION FACTOR

(a) *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.* (SRF)

A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leq$50 m)	5
C.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)	2.5
D.	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
E.	Single shear zones in competent rock (clay-free) (depth of excavation $\leq$50 m)	5.0
F.	Single shear zones in competent rock (clay-free) (depth of excavation >50 m)	2.5
G.	Loose open joints, heavily jointed or "sugar cubes," etc. (any depth)	5.0

Note: (i) Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation.

(b) *Competent rock, rock stress problems*

		σ_c/σ_1	σ_c/σ_1	(SRF)
H.	Low stress, near surface	>200	>13	2.5
J.	Medium stress	200-10	13-0.66	1.0
K.	High stress, very tight structure (usually favorable to stability, may be unfavorable for wall stability)	10-5	0.66-0.33	0.5-2
L.	Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M.	Heavy rock burst (massive rock)	<2.5	<0.16	10-20

Note: (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8\sigma_c$. When $\sigma_1/\sigma_3 >$ 10, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where : σ_c = unconfined compression strength, and σ_t = tensile strength (point load), and σ_1 and σ_3 are the major and minor principal stresses.
(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

(c) *Squeezing rock:plastic flow of incompetent rock under the influence of high rock pressure*

		(SRF)
N.	Mild squeezing rock pressure	5-10
O.	Heavy squeezing rock pressure	10-20

(d) *Swelling rock:chemical swelling inactivity depending on presence of water*

P.	Mild squeezing rock pressure	5-10
R.	Heavy squeezing rock pressure	10-15

(Sheet 3 of 3)

level of confinement in the zone of the bolts (see Figure 7-7), thus increasing the effective compressive strength of the material in the arch.

(3) Analyses of this type led Lang (1961) to formulate his empirical rules for rock bolt design, reproduced as Table 7-9. This table applies to ground conditions that

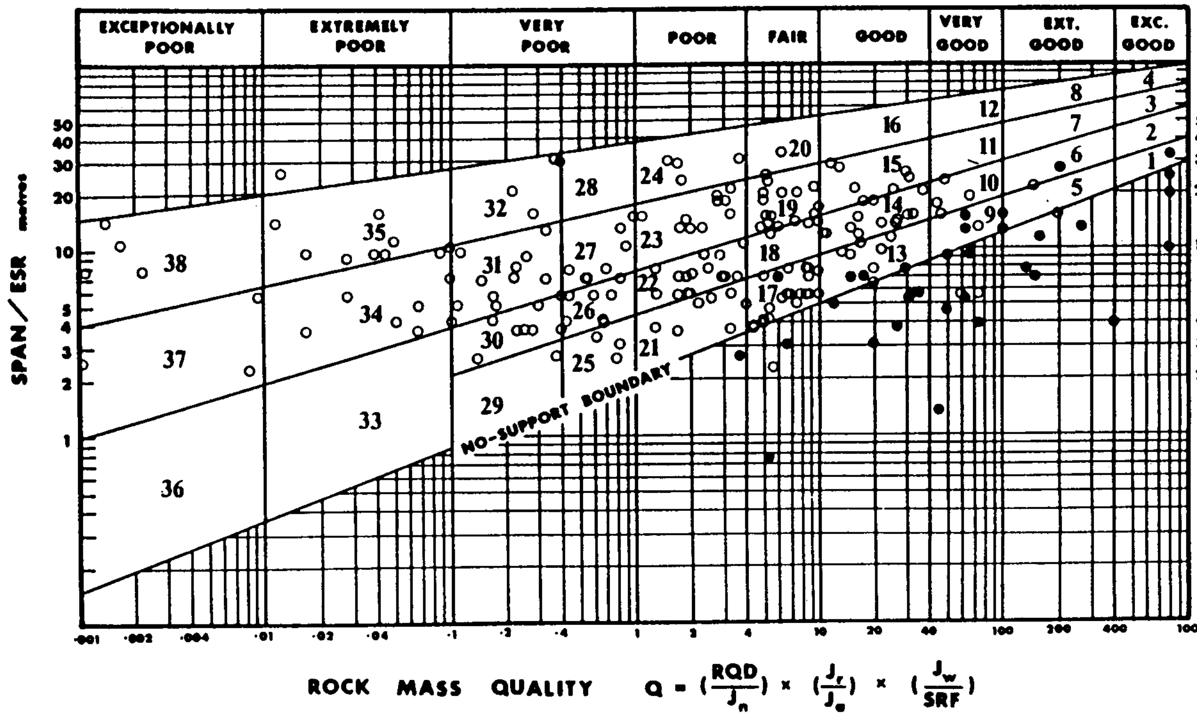


Figure 7-4. Rock support categories shown by box numbers, see Table 7-8

require more than spot bolting for ground support. Where joint spacings are so close that raveling between rock bolts is likely, the rock bolt pattern must be supplemented with wire mesh, shotcrete, or fiber-reinforced shotcrete.

b. Shotcrete analyses.

(1) The function of shotcrete in tunnel construction is to create a semistiff immediate lining on the excavated rock surface. The shotcrete must have a high initial strength for good bond to the rock surface and a high degree of ductility and toughness to absorb and block ground movement. The shotcrete, by its capacity to accept shear and bending and its bond to the rock surface, prevents the displacement of blocks of rock that can potentially fall. Shotcrete also can act as a shell and accept radial loads. It is possible to analyze all of these modes of failure only if the loads and boundary conditions are known.

(2) With the "falling block theory," the weight of a wedge of rock is assumed to load the skin of shotcrete, which can then fail by shear, diagonal tension, bonding loss, or bending (see Figure 7-8). Given the dimensions of the falling block and properties of the shotcrete, it is possible to determine the required thickness of shotcrete, using standard structural calculations.

(3) With the "arch theory," an external load is assumed, and the shotcrete shell is analyzed as an arch, with bending and compression. Where the shotcrete is held by anchors and loaded between the anchors, it may be analyzed either as a circular slab held by the anchor in the middle or as a one-way slab between rows of anchors.

(4) Neither the falling-block or the arch theory can be expected to provide anything more than crude approximations of stresses in the shotcrete, considering the dynamic environment of fresh shotcrete. When shotcrete is used in the method of sequential excavation and support such as NATM, it is possible to reproduce the construction sequence by computer analyses, including the effect of variations of shotcrete modulus and strength with time. In this fashion it is possible to estimate the load buildup in the shotcrete lining as the ground yields to additional excavation and as more layers of shotcrete are applied.

Table 7-8
Ground Support Recommendation Based on Q

Support Category	*Conditional Factors* $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$	Type of Support	Notes
1*	-	-	-	sb(utg)	-
2*	-	-	-	sb(utg)	-
3*	-	-	-	sb(utg)	-
4*	-	-	-	sb(utg)	-
5*	-	-	-	sb(utg)	-
6*	-	-	-	sb(utg)	-
7*	-	-	-	sb(utg)	-
8*	-	-	-	sb(utg)	-
Note:	The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth-wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single application of shotcrete, especially where the excavation height is ≥25 m. Future case records should differentiate categories 1 to 8.				
9	≥20	-	-	sb(utg)	-
	<20	-	-	B(utg) 2.5-3 m	-
10	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(utg) 1.5-2 m +clm	-
11*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m +clm	-
12*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m	-
13	≥10	≥1.5	-	sb(utg)	I
	≥10	<1.5	-	B(utg) 1.5-2 m	I
	<10	≥1.5	-	B(utg) 1.5-2 m	I
	<10	<1.5	-	B(utg) 1.5-2 m +S 2-3 cm	I
14	≥10	-	≥15	B(tg) 1.5-2 m +clm	I. II
	<10	-	≥15	B(tg) 1.5-2 m +S(mr) 5-10 cm	I. II
	-	-	<15	B(utg) 1.5-2 m	I. III
15	>10	-	-	B(tg) 1.5-2 m +clm	I. II. IV
	≤10	-	-	B(tg) 1.5-2 m +S(mr) 5-10 cm	I. II. IV
16* See note XII	>15	-	-	B(tg) 1.5-2 m +clm	I. V. VI
	≤15	-	-	B(tg) 1.5-2 m +S(mr) 10-15 cm	I. V. VI
17	≥30	-	-	sb(utg)	I
	≥10, ≤30	-	-	B(utg) 1-1.5 m	I
	≤10	-	≥6 m	B(utg) 1-1.5 m +S 2-3 cm	I
	<10	-	<6 m	S 2-3 cm	I

(Sheet 1 of 5)

Table 7-8 (Continued)

Support Category	*Conditional Factors* RQD/J_n	J_r/J_a	SPAN/ESR	Type of Support	Notes
18	>5	-	≥10 m	B(tg) 1-1.5 m +clm	I. III
	>5	-	<10 m	B(utg) 1-1.5 m +clm	I
	≤5	-	≥10 m	B(tg) 1-1.5 m +S 2-3 cm	I. III
	≤5	-	<10 m	B(utg) 1-1.5 m +S 2-3 cm	I
19	-	-	>20 m	B(tg) 1-2 m +S(mr) 10-15 cm	I. II. IV.
	-	-	<20 m	B(tg) 1-1.5 m +S(mr) 5-10 cm	I.II
20* See	-	-	>35	B(tg) 1-2 m +S(mr) 20-25 cm	I. V. VI
note XII	-	-	<35 m	B(tg) 1-2 m +S(mr) 10-20 cm	I.II.IV
21	≥12.5	≤0.75	-	B(utg) 1 m +S 2-3 cm	I
	<12.5	≤0.75	-	S 2.5-5 cm	I
	-	>0.75	-	B(utg) 1 m	I
	>10, <30	>1.0	-	B(utg) 1 m +clm	I
22	≤10	>1.0	-	S 2.5-7.5 cm	I
	<30	≤1.0	-	B(utg) 1 m +S(mr) 2.5-5 cm	I
	≥30	-	-	B(utg) 1 m	I
23	-	-	≥15 m	B(tg) 1-1.5 m +S(mr) 10-15 cm)	I. II. IV VII
	-	-	<15 m	B(utg) 1-1.5 m +S(mr) 5-10 cm	I
24* See	-	-	≥30 m	B(tg) 1-1.5 m +S(mr) 15-30 cm	I. V. VI.
note XII	-	-	<30 m	B(tg) 1-1.5 m +S(mr) 10-15 cm	I. II. IV
	>10	>0.5	-	B(utg) 1 m + mr or clm	I
25	≤10	>0.5	-	B(utg) 1 m +S(mr) 5 cm	I
	-	≤0.5	-	B(tg) 1 m +S(mr) 5 cm	I
26	-	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	VIII. X. XI
	-	-	-	B(tg) 1 m +S 2.5-5 cm	I. IX

(Sheet 2 of 5)

Table 7-8 (Continued)

Support Category	*Conditional Factors* RQD/J_n	J_r/J_a	SPAN/ESR	Type of Support	Notes
27	-	-	≥12 m	B(tg) 1 m +S(mr) 7.5-10 cm	I. IX
	-	-	<12 m	B(utg) 1 m +S(mr) 5-7.5 cm	I. IX
	-	-	>12 m	CCA 20-40 cm +B(tg) 1 m	VIII. X. XI
	-	-	<12 m	S(mr) 10-20 cm +B(tg) 1 m	VIII. X. XI
28* See note XII	-	-	≥30 m	B(tg) 1 m +S(mr) 30-40 cm	I. IV. V. IX
	-	-	≥20, <30 m	B(tg) 1 m +S(mr) 20-30 cm	I. II. IV. IX
	-	-	<20 m	B(tg) 1 m +S(mr) 15-20 cm	I. II. IX
	-	-	-	CCA(sr) 30-100 cm +B(tg) 1 m	IV. VIII. X. XI
29*	>5	0.25	-	B(utg) 1 m +S 2-3 cm	-
	≤5	>0.25	-	B(utg) 1 m +S(mr) 5 cm	-
	-	≤0.25	-	B(tg) 1 m +S(mr) 5 cm	-
30	≥5	-	-	B(tg) 1 m +S 2.5-5 cm	IX
	<5	-	-	S(mr) 5-7.5 cm	IX
	-	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	VIII. X. XI
31	>4	-	-	B(tg) 1 m +S(mr) 5-12.5 cm	IX
	≤4,≥1.5	-	-	S(mr) 7.5-25 cm	IX
	<1.5	-	-	CCA 20-40 cm +B(tg) 1 m	IX. XI.
	-	-	-	CCA(Sr) 30-50 cm +B(tg) 1 m	VIII. X. XI.
32 See note XII	-	-	≥20 m	B(tg) 1 m +S(mr) 40-60 cm	II. IV. IX. XI
	-	-	<20m	B(tg) 1 m +S(mr) 20-40 cm	III. IV. XI. IX.
	-	-	-	CCA(sr) 40-120 cm +B(tg) 1 m	IV. VIII. X. XI
33*	≥2	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	IX
	<2	-	-	S(mr) 5-10 cm	IX
	-	-	-	S(mr) 7.5-15 cm	VIII. X
34	≥2	≥0.25	-	B(tg) 1 m +S(mr) 5-7.5 cm	IX
	<2	≥0.25	-	S(mr) 7.5-15 cm	IX
	-	<0.25	-	S(mr) 15-25 cm	IX
	-	-	-	CCA(sr) 20-60 cm +B(tg) 1 m	VIII. X. XI

(Sheet 3 of 5)

Table 7-8 (Continued)

Support Category	*Conditional Factors* $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$	Type of Support	Notes
	-	-	≥15 m	B(tg) 1 m	II. IX. XI
				+S(mr) 30-100 cm	
35	-	-	≥15 m	CCA(sr) 60-200 cm	VIII.X.
See				+B(tg) 1 m	XI. II
note	-	-	<15 m	B(tg) 1 m	IX. III.
XII				+S(mr) 20-75 cm	XI.
			<15 m	CCA(sr) 40-150 cm	VII. X.
				+B(tg) 1 m	XI. III
	-	-	-	S(mr) 10-20 cm	IX
36*	-	-	-	S(mr) 10-20 cm	VIII. X.
				+B(tg) 0.5-1.0 m	XI
37	-	-	-	S(mr) 10-20 cm	IX
				S(mr) 20-60 cm	VIII. X.
				+B(tg) 0.5-1.0 m	XI
			≥10 m	CCA(sr)100-300 cm	IX
38	-	-	≥10 m	CCA(sr)100-300 cm	VIII. X.
See				+B(tg) 0.5-1.0 m	II. XI
note	-	-	<10 m	S(mr) 70-200 cm	IX
XIII	-		<10 m	S(mr) 70-200 cm	VII. X.
				+B(tg) 1 m	III. XI

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

Key to Support Tables:

sb	=	spot bolting
B	=	systematic bolting
(utg)	=	untensioned, grouted
(tg)	=	tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S	=	shotcrete
(mr)	=	mesh reinforced
clm	=	chain link mesh
CCA	=	cast concrete arch
(sr)	=	steel reinforced

Supplementary Notes by BARTON, LIEN *and* LUNDE

I. For cases of heavy bursting or "popping," tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.

II. Several bolt lengths often used in same excavation, i.e., 3, 5, and 7 m.

III. Several bolt lengths often used in same excavation, i.e., 2, 3, and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2-4 m.

V. Several bolt lengths often used in same excavation, i.e., 6, 8, and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4-6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain-link mesh, and a free span concrete arch roof (25-40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

(Sheet 4 of 5)

Table 7-8 (Concluded)

IX.	Cases not involving swelling clay or squeezing rock.
X.	Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.
XI.	According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e., >1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e., RQD/J_n < 1.5, for example, a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete, but it may not be effective when RQD/J_n < 1.5 or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick-setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.
XII.	For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (SPAN/ESR > 15 m only).
XIII.	Multiple drift method usually needed during excavation and support of arch, walls, and floor in cases of heavy squeezing. Category 38 (SPAN/ESR > 10 m only).
Supplementary notes by HOEK *and* BROWN *(1980)*	
A.	Chain-link mesh is sometimes used to catch small pieces of rock that can become loose with time. It should be attached to the rock at intervals of between 1 and 1.5 m, and short grouted pins can be used between bolts. Galvanized chain-link mesh should be used where it is intended to be permanent, e.g., in an underground powerhouse.
B.	Weldmesh, consisting of steel wires set on a square pattern and welded at each intersection, should be used for the reinforcement of shotcrete since it allows easy access of the shotcrete to the rock. Chain-link mesh should never be used for this purpose since the shotcrete cannot penetrate all the spaces between the wires and air pockets are formed with consequent rusting of the wire. When choosing weldmesh, it is important that the mesh can be handled by one or two men working from the top of a high-lift vehicle and hence the mesh should not be too heavy. Typically, 4.2-mm wires set at 100-mm intervals (designated 100 by 100 by 4.2 weldmesh) are used for reinforcing shotcrete.
C.	In poorer quality rock, the use of untensioned grouted dowels as recommended by Barton, Lien, and Lunde (1974) depends upon immediate installation of these reinforcing elements behind the face. This depends upon integrating the support drilling and installation into the drill-blast-muck cycle, and many non-Scandinavian contractors are not prepared to consider this system. When it is impossible to ensure that untensioned grouted dowels are going to be installed immediately behind the face, consideration should be given to using tensioned rock bolts that can be grouted at a later stage. This ensures that support is available during the critical excavation stage.
D.	Many contractors would consider that a 200-mm-thick cast concrete arch is too difficult to construct because there is not enough room between the shutter and the surrounding rock to permit easy access for placing concrete and using vibrators. The USACE has historically used 10 in. (254 mm) as a normal minimum, while some contractors prefer 300 mm.
E.	Barton, Lien, and Lunde (1974) suggest shotcrete thicknesses of up to 2 m. This would require many separate applications, and many contractors would regard shotcrete thicknesses of this magnitude as both impractical and uneconomical, preferring to cast concrete arches instead. A strong argument in favor of shotcrete is that it can be placed very close to the face and hence can be used to provide early support in poor quality rock masses. Many contractors would argue that a 50- to 100-mm layer is generally sufficient for this purpose, particularly when used in conjunction with tensioned rock bolts as indicated by Barton, Lion, and Lunde (1974) and that the placing of a cast concrete lining at a later stage would be a more effective way to tackle the problem. Obviously, the final choice will depend upon the unit rates for concreting and shotcreting offered by the contractor and, if shotcrete is cheaper, upon a practical demonstration by the contractor that he can actually place shotcrete to this thickness.
	In North America, the use of concrete or shotcrete linings of up to 2 m thick would be considered unusual, and a combination of heavy steel stets and concrete would normally be used to achieve the high support pressures required in very poor ground.
Supplementary note	
	Untensioned, grouted rock bolts are recommended in several support categories. At the time when Barton, Lien, and Lunde proposed their guide for support measures, the friction-anchored rock bolts were not yet available. Under appropriate circumstances, friction dowels are relatively inexpensive alternatives for initial, temporary ground-support application.

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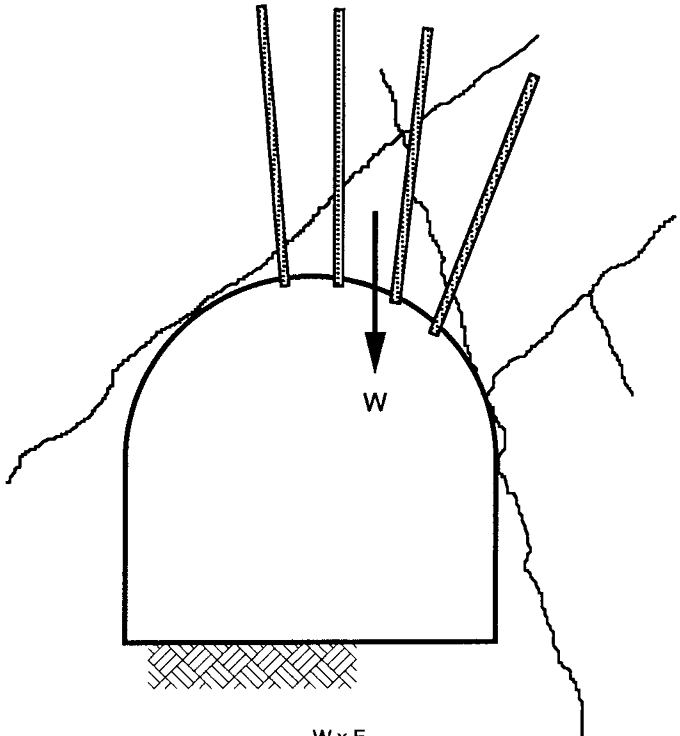
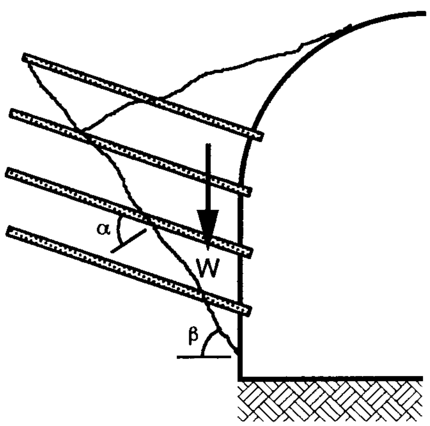


$$N = \frac{W \times F}{B}$$

N = Number of bolts (dowels)
W = Weight of wedge
F = Safety factor (1.5 to 3.0)
φ = Friction angle of sliding surface
c = Cohesion of sliding surface
A = Area of sliding surface
B = Load bearing capacity of bolt (dowel)



$$N = \frac{W\,(F \sin\beta - \cos\beta \tan\varphi) - cA}{B\,(\cos\alpha \tan\varphi + F \sin\alpha)}$$

Figure 7-5. Gravity wedge analyses to determine anchor loads and orientations

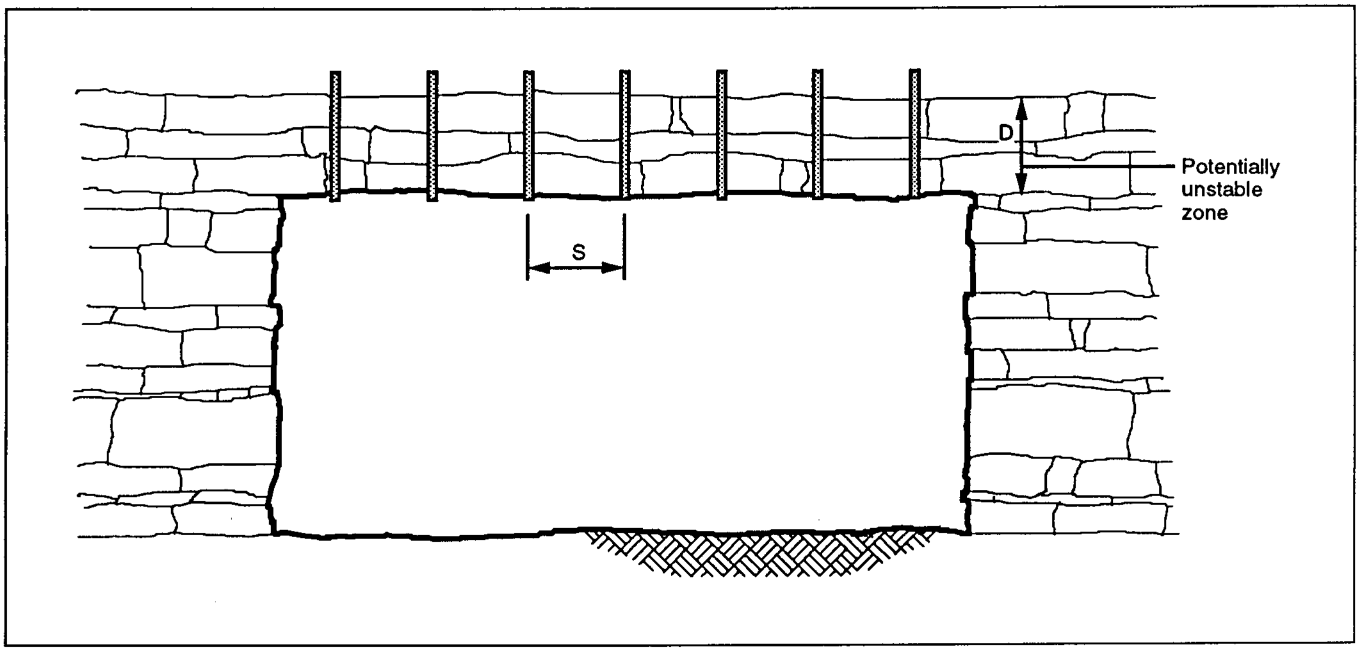


Figure 7-6. Reinforced roof beam

7-4. Design of Steel Ribs and Lattice Girders

In today's tunneling, steel ribs are still used for many purposes. This subsection deals with the selection and design of steel rib supports and lattice girders.

a. Use of steel ribs and lattice girders.

(1) Steel ribs are usually made of straight or bent I-beams or H-beams, bolted together to form a circular or pitched arch with straight, vertical side supports (legs), or a true horseshoe shape with curved legs, sometimes with a straight or curved horizontal invert strut. Full-circle steel sets are also common. Structural shapes other than I- or H-beams have also been used.

(2) Steel sets are most often used as ground support near tunnel portals and at intersections, for TBM starter tunnels, and in poor ground in blasted tunnels. Steel sets are also used in TBM tunnels in poor ground when a reaction platform for propulsion is required. The traditional blocking consists of timber blocks and wedges, tightly installed between the sets and the rock, with an attempt to prestress the set. Timbers not essential for ground support are generally removed before placing a final, cast-in-place concrete lining. Recently, blocking made of concrete or steel is often specified. This method is more difficult to work with, and a more flexible method consists of using special bags pumped full of concrete. These bags will accommodate themselves to the shape of the rock as excavated and form a firm contact with the rock.

(3) Shotcrete is also used as blocking material. When well placed, shotcrete fills the space between the steel rib and the rock and is thus superior to other methods of blocking by providing for a uniform interaction between the ground and the support. Care must be exercised to fill all the voids behind each rib.

(4) Lattice girders offer similar moment capacity at a lower weight than comparable steel ribs. They are easier to handle and erect. Their open lattice permits shotcrete to be placed with little or no voids in the shadows behind the steel structure, thus forming a composite structure. They can also be used together with dowels, spiling, and wire mesh, and (see Figure 5-19) as the final lining.

b. Design of blocked ribs.

(1) The still-popular classical text provided in Proctor and White (1946) is the best guide to the design of steel ribs installed with blocking. The designer is referred to this text for details of design and several design charts and to the available commercial literature for the design of connections and other details. The basic theory behind the classical method of rib design is that the flexibility of the steel rib/timber blocking system permits essentially complete load redistribution. Vertical loads transferred

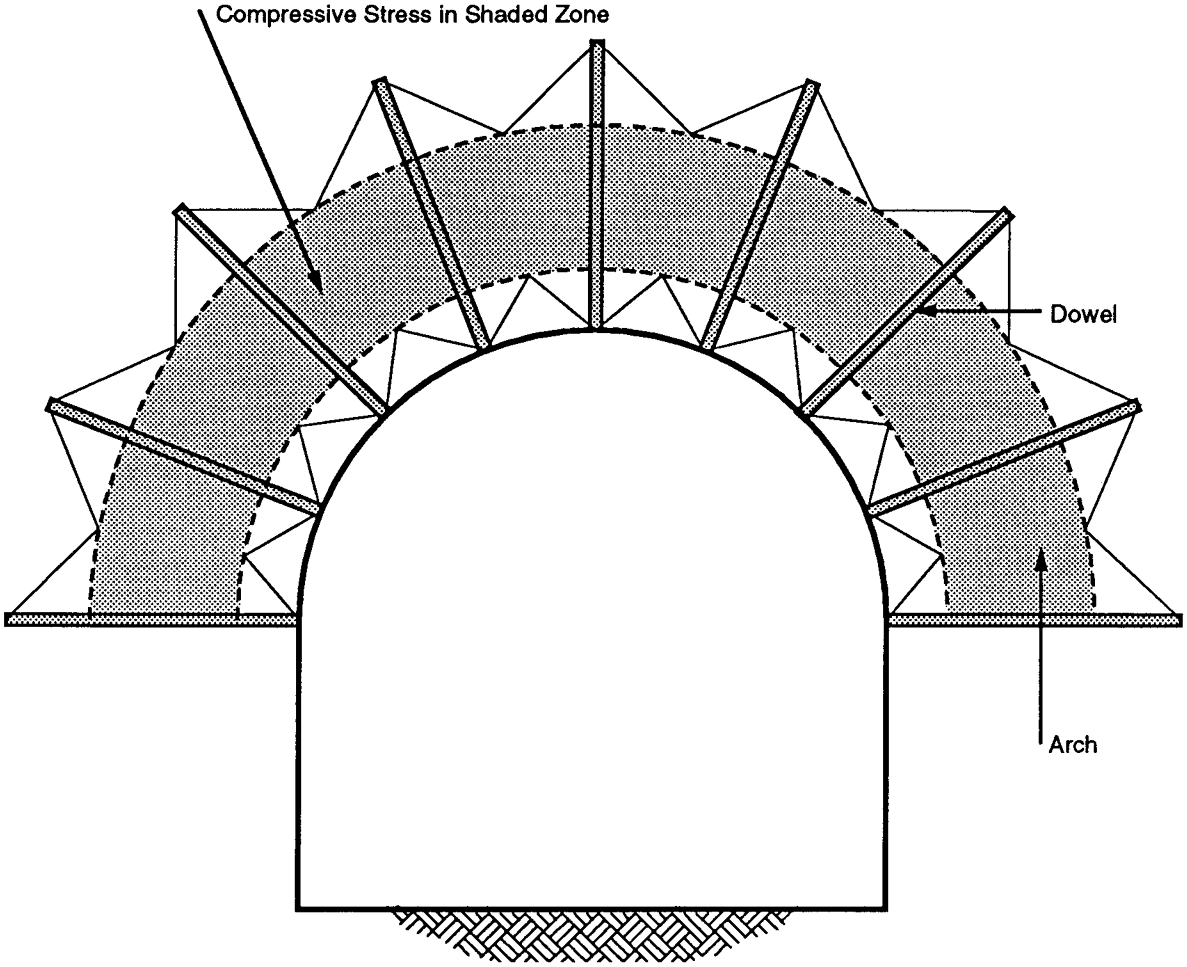


Figure 7-7. Reinforced roof arch

through the blocking cause a deformation sufficient to generate reactions along the sides, such that loads around the arch become essentially uniform. Loads at an angle with vertical have the same effect. Thus, the combined loads result in a uniform thrust in the rib (T), and the maximum moment occurs at blocking points and at points in the middle between blocking points. If the rib was assumed to be pinned at the blocking points, the moment would be equal to the thrust multiplied by the rise of the arc (h) between the blocking points ($M_t = Th$). In fact, the rib is continuous, and there is a moment (M_b) at the blocking points. The maximum moment, then, is $M_m = M_t - M_b$.

(2) If the arch is continuous, fixed at both ends, and bears against equally spaced blocking points, then the maximum moment occurs at blocking points and is approximately $M_{max} = M_b = 0.67\ M_t = 0.67\ Th$. If the arch is hinged at both ends, the maximum moment is 0.86 Th.

(3) When the arch is fixed at the top of a straight leg, the moment in the leg is 0.67 Th, reducing to zero at the bottom, assumed as a hinge. When there are significant side pressures on the legs, the leg moments become larger, the legs must be prevented from kicking in, and arched (horseshoe) legs are often used, together with invert struts.

Table 7-9
Empirical Design Recommendations

Parameter		*Empirical Rule*
Minimum length and maximum spacing		
Minimum length		Greatest of
	(a)	2 x bolt spacing
	(b)	3 x thickness of critical and potentially unstable rock blocks (Note 1)
	(c)	For elements above the springline: spans <6 m: 0.5 x span spans between 18 and 30 m: 0.25 x span
	(d)	For elements below the springline: height <18 m: as (c) above height >18 m: 0.2 x height
Maximum spacing		Least of:
	(a)	0.5 x bolt length
	(b)	1.5 x width of critical and potentially unstable rock blocks (Note 1)
	(c)	2.0 m (Note 2)
Minimum spacing		0.9 to 1.2 m
Minimum average confining pressure		
Minimum average confining pressure at yield point of elements (Note 3)		Greatest of
	(a)	Above springline: *either* pressure = vertical rock load of 0.2 x opening width or 40 kN/m^2
	(b)	Below springline: *either* pressure = vertical rock load of 0.1 x opening height of 40 kNm^2
	(c)	At intersections: 2 x confining pressure determined above (Note 4)

Notes:
1. Where joint spacing is close and span relatively large, the superposition of two reinforcement patterns may be appropriate (e.g., long heavy elements on wide centers to support the span, and shorter, lighter bolts on closer centers to stabilize the surface against raveling).
2. Greater spacing than 2.0 m makes attachment of surface support elements (e.g., weldmesh or chain-link mesh) difficult.
3. Assuming the elements behave in a ductile manner.
4. This reinforcement should be installed from the first opening excavated prior to forming the intersection. Stress concentrations are generally higher at intersections, and rock blocks are free to move toward both openings.

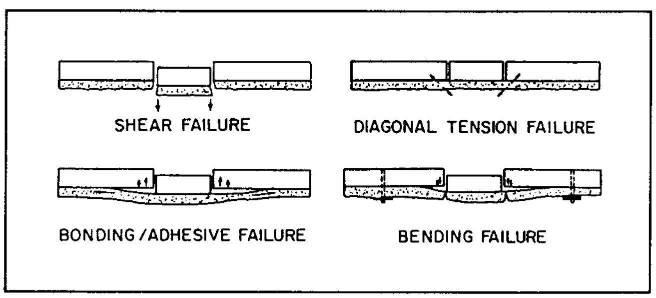


Figure 7-8. Shotcrete failure modes

With very large side pressure, such as in squeezing ground, a full circular shape is used.

c. Lattice girders with continuous blocking.

(1) The theory for blocked arches works adequately for curved structural elements if the blocking is able to deform in response to applied loads, provided the arch transmits a thrust and moment to the end points of the arch. With continuous blocking by shotcrete, however, the

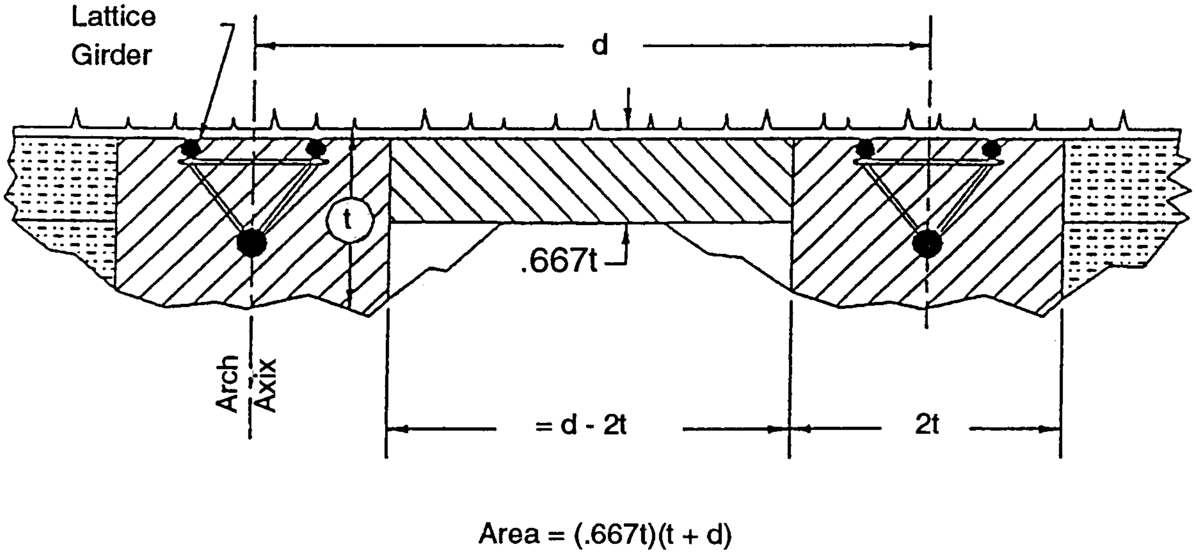


Figure 7-9. Estimation of cross section for shotcrete-encased lattice girders

blocking does not yield significantly once it has set and load redistribution is a function of excavation and installation sequences. Moments in the composite structure should preferably be estimated using one of the methods described in Chapter 9. To estimate moments for sequential excavation and support, where the ground support for a tunnel station may be constructed in stages, finite element or finite difference methods are preferred. These analyses should ideally incorporate at least the following features:

- Unloading of the rock due to excavation.
- Application of ground support.
  - First shotcrete application.
  - Lattice girder installation.
  - Subsequent shotcrete application.
  - Other ground support (dowels, etc.) as applicable.
- Increase in shotcrete modulus with time as it cures.
- Repeat for all partial face excavation sequences until lining closure is achieved.

(2) These types of analysis only yield approximate results. However, they are useful to study variations in construction sequences, locations of maximum moments and thrusts, and effects of variations of material properties and in situ stress.

(3) Stresses in composite lattice girder and shotcrete linings can be analyzed in a manner similar to reinforced concrete subjected to thrust and bending (see Chapter 9). Figure 7-9 shows an approximation of the typical application of lattice girders and shotcrete. The moment capacity analysis should be performed using the applicable shotcrete strength at the time considered in the analysis.